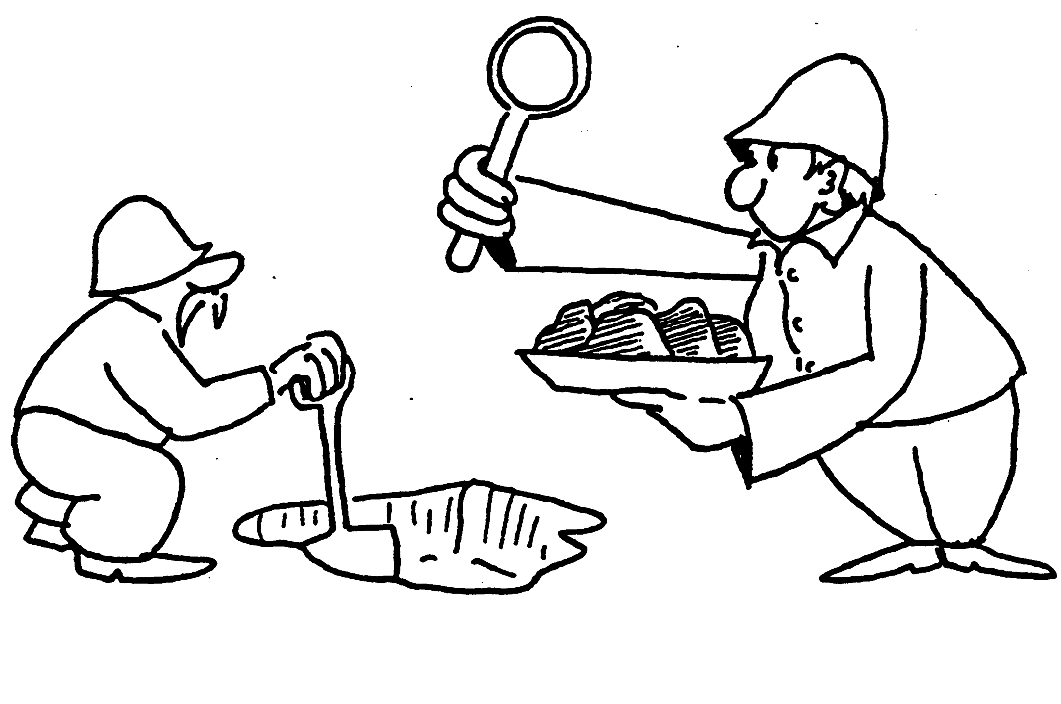
# SOIL CLASSIFICATION AND PROPERTIES



## SOIL CLASSIFICATION AND PROPERTIES

To engineer an adequate earth retaining system for a trench or excavation it is first necessary to identify the material (soil) in which the excavation is to be made. The basic load upon an earth retaining system is caused by the resulting lateral pressure of the retained soil. Soils can be quite different. It is these differences that result in variations in the lateral earth loads or pressures.

In general, there are two types of soils; cohesionless soils (sand and gravel) and cohesive soils (clays). Silts, depending on plasticity, may or may not be considered cohesionless. Natural soils areusually between these two extremes. Unusual soil types such as organic peat and permafrost conditions are not addressed in this manual.

A soil classification defines what soil is comprised of (silty sand for example). Various classification systems have been established. The Department of Transportation prefers the use of the ASTM Unified Classification System. This system was initially developed by the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation in 1952. The classification system is discussed in detail in the California Department of Transportation Materials Manual Volume VI 1973. Geotechnical Engineers and Contractors will not always use the same classification system and word description. If enough of the soil properties are known, a soil can be described-or classified properly. in the Unified Soil Classification System.

Classification is only the first part of a soil description. There are various characteristics or properties that must be known in order to predict the effect of a soil on an earth retaining system. These are the Engineering Properties of the soil. Standard means of measuring and determining these properties have been developed.

Soils Investigation usually consists of obtaining representative soil samples, performing tests, and summarizing the data. Additional pertinent information such as ground-water conditions, recommendations for an equivalent fluids oil pressure (Kw) and shape of pressure distribution diagrams may also be included.

Soil shear strength is of primary concern in trenching and shoring work. One of the fundamental relationships governing soil shear strength was first recognized by Coulomb and can be expressed as follows:

$$s = c' + \sigma' \tan\phi'$$
$$= c' + (\sigma - u)\tan\phi'$$

where $s$ = soil shear strenght
$c'$ = effective cohesion intercept
$\sigma'$ = effective normal stress (stress causing soil particles to press together)
$\phi'$ = effective angle of internal friction
$\sigma$ = total normal stress
$u$ = pore water pressure

In general, this relationship between shear strength and normal stress is not linear for large stress ranges. Therefore the shear strength parameters c' and $\sigma'$ should be defined for narrow stress ranges.

Depending on soil permeability and the degree of saturation, the presence of water will tend to prevent a soil from changing volume when it is loaded. Without volume change, there can be no change in effective normal stress and therefore no change in soil shear strength. When a soil is saturated, a change in loading will produce a change in pore water pressure which is in excess of thehydrostatic pore water pressure. Excess pore water pressure will not necessarily be positive, and may, depending on soil stress history and loading conditions, be negative.

If a soil has high permeability, such as a coarse sand, the excess pore water pressure will dissipate almost instantly and there will be an instantaneous change in shear strength.

If a soil has low permeability, a clay for example, then excess pore water pressure will dissipate very slowly and no change in shear strength will be observed for quite sometime. Because the strength of saturated impervious fine grained soils changes slowly with externally applied pressures, their strength can sometimes be expressed as:

$$s = s_u$$

where $s_u$ is the undrained shear strength.

These fine-grained types of soil, because their shear strength is initially indifferent to confining pressures, are often said to derive their strength through "cohesion" and are many times referred to as "cohesive soils."

In cohesive soils, excess pore pressure will reach zero over a period of time as the soil either consolidates or swells (depending respectively on whether the soil has been loaded or unloaded.) Trenching and shoring work often creates situations where soil loading is reduced - an excavation for example. A fine-grained soil in this situation will tend to expand and has the potential to lose shear strength over time.

Soil permeability, drainage and loading conditions, and degree of saturation greatly effect the pore pressures generated when soils are loaded, which in turn significantly affect Soil shear strength. Therefore, these factors require careful consideration when planning or evaluating a soil testing program to estimate shear strength parameters for use in analysis.

A few helpful TABLES and FIGURES are included in this section listing various soil properties and relationships.

An essential value for the determination of some of the soil relationships described in TABLE 7 is the specificgravity (G) of the soil. The specific gravity (G) of a soil may be satisfactorily estimated in accordance with the following:

| Soil Tvpe | Specific Gravitv (G) |
|---|---|
| Sands and Gravels | 2.65 - 2.68 |
| Inorganic Silt | 2.62 - 2.68 |
| Organic Clay | 2.58 - 2.65 |
| Inorganic Clay | 2.68 - 2.75 |

# VOLUME AND WEIGHT RELATIONSHIPS

| | Property | | Saturated sample ($W_s$, $W_w$, G, are known) | Unsaturated sample ($W_s$, $W_w$, G, V are known) | Supplementary formulas relating measured and computed factors | | | |
|---|---|---|---|---|---|---|---|---|
| Volume components | $V_s$ | volume of solids | $\frac{W_s}{G\gamma_w}$ | | $V-(V_a+V_w)$ | $V(1-n)$ | $\frac{V}{(1+e)}$ | $V_v/e$ |
| | $V_w$ | volume of water | $W_w/\gamma_w{}^*$ | | $V_v - V_a$ | $SV_v$ | $\frac{SVe}{(1+e)}$ | $SV_se$ |
| | $V_a$ | volume of air or gas | zero | $V-(V_s+V_w)$ | $V_v - V_w$ | $(1-S)V_v$ | $\frac{(1-S)Ve}{(1+e)}$ | $(1-S)V_se$ |
| | $V_v$ | volume of voids | $W_w/\gamma_w{}^*$ | $V - \frac{W_s}{G\gamma_w}$ | $V - V_s$ | $\frac{V_sn}{1-n}$ | $\frac{Ve}{(1+e)}$ | $V_se$ |
| | $V$ | total volume of sample | $V_s + V_w$ | measured | $V_s + V_a + V_w$ | $\frac{V_s}{1-n}$ | $V_s(1+e)$ | $\frac{V_v(1+e)}{e}$ |
| | $n$ | porosity | $V_v/V$ | | $1 - V_s/V$ | $1 - \frac{W_s}{GV\gamma_w}$ | $\frac{e}{1+e}$ | |
| | $e$ | void ratio | $V_v/V_s$ | | $V/V_s - 1$ | $\frac{GV\gamma_w}{W_s} - 1$ | $\frac{W_wG}{W_sS}$ | $\frac{n}{1-n}$ ; $\frac{wG}{S}$ |
| Weights for specific sample | $W_s$ | weight of solids | measured | | $\frac{W_T}{(1+w)}$ | $GV\gamma_w(1-n)$ | $\frac{W_wG}{eS}$ | |
| | $W_w$ | weight of water | measured | | $wW_s$ | $S\gamma_wV_v$ | $\frac{eW_sS}{G}$ | |
| | $W_t$ | total weight of sample | $W_s + W_w$ | | $W_s(1+w)$ | | | |
| Weights for sample of unit volume | $\gamma_D$ | dry unit weight | $\frac{W_s}{V_s+V_w}$ | $W_s/V$ | $\frac{W_T}{V(1+w)}$ | $\frac{G\gamma_w}{(1+e)}$ | $\frac{G\gamma_w}{1+wG/S}$ | |
| | $\gamma_T$ | wet unit weight | $\frac{W_s+W_w}{V_s+V_w}$ | $\frac{W_s+W_w}{V}$ | $W_T/V$ | $\frac{(G+Se)\gamma_w}{(1+e)}$ | $\frac{(1+w)\gamma_w}{w/S+1/G}$ | |
| | $\gamma_{SAT}$ | saturated unit weight | $\frac{W_s+W_w}{V_s+V_w}$ | $\frac{W_s+V_v\gamma_w}{V}$ | $W_s/V + \left(\frac{e}{1+e}\right)\gamma_w$ | $\frac{(G+e)\gamma_w}{(1+e)}$ | $\frac{(1+w)\gamma_w}{w+1/G}$ | |
| | $\gamma_{SUB}$ | submerged (buoyant) unit weight | $\gamma_{SAT} - \gamma_w{}^*$ | | $W_s/V - \left(\frac{1}{1+e}\right)\gamma_w{}^*$ | $\left(\frac{G+e}{1+e} - 1\right)\gamma_w{}^*$ | $\left(\frac{1-1/G}{w+1/G}\right)\gamma_w{}^*$ | |
| Combined relations | $w$ | moisture content | $W_w/W_s$ | | $W_t/W_s - 1$ | $\frac{Se}{G}$ | $S\left[\frac{\gamma_w{}^*}{\gamma_D} - \frac{1}{G}\right]$ | |
| | $S$ | degree of saturation | 1.00 | $V_w/V_v$ | $\frac{W_w}{V_v\gamma_w{}^*}$ | $\frac{wG}{e}$ | $\frac{w}{\left[\frac{\gamma_w{}^*}{\gamma_D} - \frac{1}{G}\right]}$ | |
| | $G$ | specific gravity | $\frac{W_s}{V_s\gamma_w}$ | | $\frac{Se}{w}$ | | | |

Notes.—$\gamma_w$ is unit weight of water, which equals 62.4 pcf for fresh water and 64 pcf for sea water (1.00 and 1.025 gm/cc). (Where noted with * the actual unit weight of water surrounding the soil is used.) In other cases use 62.4 pcf.
Values of (w) and (s) are used as decimal numbers.

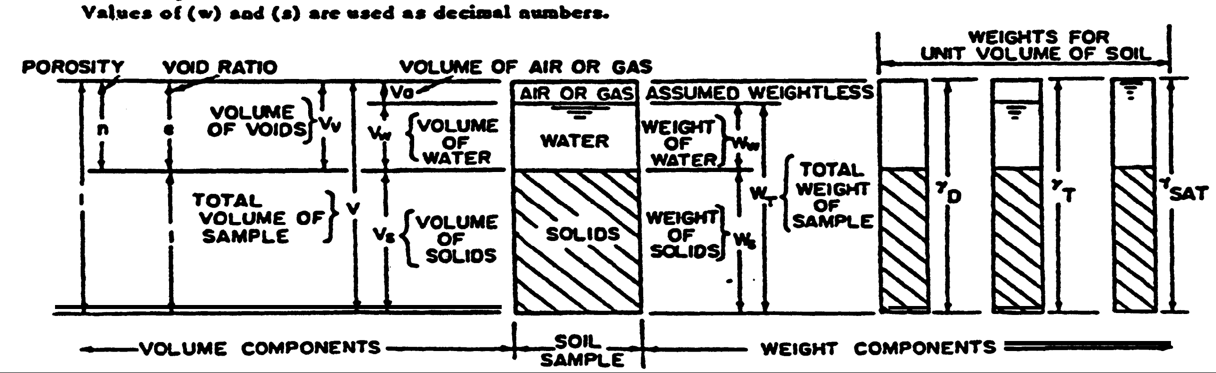
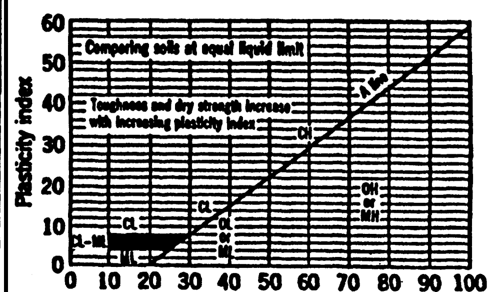



TABLE 7

# UNIFIED SOIL CLASSIFICATION CHART

| Major divisions | | | Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights) | Group Symbols[a] | Typical Names | Information Required for Describing Soils | Laboratory Classification Criteria | |
|---|---|---|---|---|---|---|---|---|
| Coarse-grained soils. More than half of material is *larger* than No. 200 sieve size[b] (The No. 200 sieve size is about the smallest particle visible to naked eye) | Gravels. More than half of coarse fraction is larger than No. 7 sieve size (For visual classification, the ¼ in. size may be used as equivalent to the No. 7 sieve size) | Clean gravels (Little or no fines) | Wide range in grain size and substantial amounts of all intermediate particle sizes | *GW* | Well graded gravels, gravel-sand mixtures, little or no fines | Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses | Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols | $C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3 |
| | | | Predominantly one size or a range of sizes with some intermediate sizes missing | *GP* | Poorly graded gravels, gravel-sand mixtures, little or no fines | | | Not meeting all gradation requirements for *GW* |
| | | Gravels with fines (appreciable amount of fines) | Nonplastic fines (for identification procedures see *ML* below) | *GM* | Silty gravels, poorly graded gravel-sand-silt mixtures | | | Atterberg limits below "A" line, or *PI* less than 4. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols |
| | | | Plastic fines (for identification procedures, see *CL* below) | *GC* | Clayey gravels, poorly graded gravel-sand-clay mixtures | For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics | | Atterberg limits above "A" line, with *PI* greater than 7 |
| | Sands. More than half of coarse fraction is smaller than No. 7 sieve size | Clean sands (Little or no fines) | Wide range in grain sizes and substantial amounts of all intermediate particle sizes | *SW* | Well graded sands, gravelly sands, little or no fines | Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; *(SM)* | | $C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3 |
| | | | Predominantly one size or a range of sizes with some intermediate sizes missing | *SP* | Poorly graded sands, gravelly sands, little or no fines | | | Not meeting all gradation requirements for *SW* |
| | | Sands with fines (appreciable amount of fines) | Nonplastic fines (for identification procedures, see *ML* below) | *SM* | Silty sands, poorly graded sand-silt mixtures | | | Atterberg limits below "A" line or *PI* less than 5. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols |
| | | | Plastic fines (for identification procedures, see *CL* below) | *SC* | Clayey sands, poorly graded sand-clay mixtures | | | Atterberg limits below "A" line with *PI* greater than 7 |

| Major divisions | | Dry Strength (crushing characteristics) | Dilatancy (reaction to shaking) | Toughness (consistency near plastic limit) | Group Symbols[a] | Typical Names | Information Required for Describing Soils |
|---|---|---|---|---|---|---|---|
| Fine-grained soils. More than half of material is *smaller* than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to naked eye) | Identification Procedures on Fraction Smaller than No. 40 Sieve Size | | | | | | |
| | Silts and clays liquid limit less than 50 | None to slight | Quick to slow | None | *ML* | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity | Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses |
| | | Medium to high | None to very slow | Medium | *CL* | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays | |
| | | Slight to medium | Slow | Slight | *OL* | Organic silts and organic silt-clays of low plasticity | For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions |
| | Silts and clays liquid limit greater than 50 | Slight to medium | Slow to none | Slight to medium | *MH* | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts | |
| | | High to very high | None | High | *CH* | Inorganic clays of high plasticity, fat clays | Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; *(ML)* |
| | | Medium to high | None to very slow | Slight to medium | *OH* | Organic clays of medium to high plasticity | |
| Highly Organic Soils | | Readily identified by colour, odour, spongy feel and frequently by fibrous texture | | | *Pt* | Peat and other highly organic soils | |



Plasticity chart
for laboratory classification of fine grained soils

TABLE 8

## SOIL PROPERTY NOMENCLATURE

| | |
|---|---|
| $\gamma_{dry}$ | = Dry Unit Weight (pcf) |
| $\gamma_{sat}$ | = Saturated Unit Weight (pcf) |
| $\gamma_{sub}$ | = Submerged Unit Weight (pcf) |
| $\gamma_w$ | = Unit Weight of Water (pcf) |
| $\gamma_{sat}$ | $= \gamma_{sub} + \gamma_w$ |
| $\gamma_{sub}$ | $\approx 0.6\gamma_{dry} \approx \gamma_d(1 - 1/G)$ [Assuming G = 2.65] |
| $\gamma_{sat}$ | $\approx 0.6\gamma_{dry} + \gamma_w$ |
| $q_u$ | = Unconfined compressive strength (psf) |
| $s_u$ | = Undrained Shear Strength = $q_u/2$ (psf) for the unconfined compression test. |
| c | = Cohesion intercept: the component of soil shear strength which is independent of the force normal to the shear plane. |
| $\phi$ | = Angle of Internal Friction (degree): represents the frictional component of soil shear strength, dependent on the forces pushing the particles together. |
| $\theta$ | = Angle of Repose (degrees) (The angle at which a soil will stand unsupported). |
| $w_p$ | = PLASTIC LIMIT: This is the moisture content at which the soil mass ceases to be plastic; it will become brittle or crumbly without any further reduction in moisture content. |
| $w_L$ | = LIQUID LIMIT: The water content at which the soil has such a small shear strength that it flows to close a grove of standard width when jarred in a specified manner. |

TABLE 9

Test Borings permit classification and consistency determinations of underlying soils. Soils usually change at various levels or depths. Sometimes it is necessary to identify soils below the bottom of the proposed excavation. Soils samples for testing may be extracted and the presence and the elevation of ground water determined.

The Standard Penetration Test (SPT) is a means of retrieving, from the bottom of a bore hole, a disturbed sample of soil for visual classification or index testing. The number of hammer blows used to drive the sampler provides an indication of thedensity of a granular soil or the consistency of a fine-grained soil. Empirical relationships can then be used which estimated soil friction angle ($\phi$)from density of granular soils and unconfined compressive strength ($q_u$) from consistency of fine-grained soils.

Standard test procedures have been developed to obtain the properties from field samples. Some of these tests will be performed in the field and the remainder in a soils laboratory. For detailed information on soils investigation procedures and tests, see the Department of Transportation Materials Manual Volume VI, 1973.

There are several sources of soils information usually available to the Contractor and the Engineer. When structures are included in a contract there will be a report prepared by the Transportation Materials and Research Laboratory Engineering Geology Branch. The "Log of Test Borings", which is included as part of the contract plans is from this report. The soil is classified and its density or hardness at various elevations is determined. Moisture content and ground water conditions may also be given. If the borings are within reasonable distance of the proposed trench work they may serve as a guide to review or confirmthe soils data submitted with the shoring or excavation plans. A sample of a Log of Test Borings is shown in FIGURE 5.

Soils investigations may also be made by District Materials and Headquarters Laboratories. Numerous properties can be developed beyond those normally shown on the Log of Test Borings. This additional soils information is a part of the project materials -report. An example portion of a reporting form is shown in FIGURE 6.

Material reports are available on request. The Contractor should be informed of the availability of soils information at the pre-job conference. Observation of adjacent work in similar material by both the engineer and the Contractor during such operations as pile driving and excavations will often supply useful soils information.

The Contractor may elect, or find it necessary, to have a soils investigation performed. In this case the soils information or report will be furnished to the Engineer as a part of the supporting data accompanying the shoring plans.

It is recommended that exploratory borings be made and soil properties determined for unusual conditions such as a very deep excavation, work adjacent to buildings, and known areas of potentially unstable ground. This is especially critical when ground water level is close to the surface. Materials that require special consideration include existing and former tidal flats, estuaries, marshes, alluvial flats, andground reclaimed by fill.

Soil test results need to be used with caution. Soil test reports from many soils laboratories or similar sources will include safety factors incorporated in the reported results. Other soil test data may not include safety factor considerations. The soil properties shown in FIGURE 6 for example, are direct result values which do not contain appropriate factors of safety.

Factors which the engineer will consider when assigning strength parameters to a soil include:

- the method with which soil shear strengthwas determined.
- the variability of subsurface-profile.
- the number and distribution of shear strength tests.

Of these factors only the first can be addressed here, the others must be dealt with on a site specific basis.

Many ways of evaluating soil shear strength have been developed. Not all methods are equally precise, therefore one needs to consider the source of the shear strength data when making an evaluation of a proposed trenching or shoring system. The following is a guide for judging the reliability of soil shear strength parameters:

| Test Method | Course-grained soil | Fine-grained soil |
|---|---|---|
| Triaxial compression test | very good* | very good |
| Unconfined compression test | mot applicable | very good |
| Direct shear test | good* | fair |
| Vane shear test | not applicable | good |
| Cone penetration test (CPT) | fair | fair |
| Pocket penetrometer | not applicable | fair |
| Standard penetration test (SPT) | fair | poor |

*Recovery of undisturbed samples can be difficult.

Additional information concerning soils testing may be found in Appendix B.

Unstable ground conditions may also be encountered in areas underlain by soils developed in-situ from weathering of rock and in deeply weathered and sheared rock. Adversely oriented bedding, fracturing and jointing planes of shear zones which are inclined towards the excavation should be investigated.

Instability of excavation walls can also occur in certain geologic formations such as clays and shales that are subject to cracking and spalling upon exposure to the atmosphere and to swelling when saturated with water. Excavating in such materials requires protection of the excavation walls to help retain natural moisture content and thus prevent cracking, spalling, and eventual wall instability. If cracking does occur, water should be prevented from seeping into the cracks.

A term that comes up frequently in trenching work is "running ground." It is referred to in the Construction Safety Orders and is criteria for more restrictive requirements for a shoring system. A running ground is defined as a soil that cannot stand by itself even for a short term, and is the dynamic state of actual failure or cave-in. Running soil will have little shear strength and will flow with virtually no angle of response in an unsupported condition. A mud under pressure which flows is an example of running ground. For running ground conditions, the full dry weight or the saturated unit weight of the material has to be resisted. The angle of internal friction ($\phi$)and the cohesive value, are both zero. The shoring system wall in contact with the material must be solid. Running soil is the most adverse soil conditions that can be encountered. The soils investigation should state if a soil is known to be running.

Quick sand is a type of running soil. It occurs in cohesionless soil when the force of the upward flow of water is sufficient to make the soil bouyant and there by prevent grain interlocking. The soil grains are suspended in the water. *A* quick condition can be developed by adverse water flow. It may best be stabilized when the trench is dewatered. Quick conditions can occur in silt as we11 as in sand.

| Major Divisions | | | Group Symbols | Typical Names |
|---|---|---|---|---|
| Coarse-grained Soils | Gravels | Clean gravels (Little or no fines) | GW | Well-graded gravels, gravel-sand mixtures, little or no fines |
| | | | GP | Poorly graded gravels, gravel-sand mixtures, little or no fines |
| | | Gravels with fines (Appreciable amount of fines) | GM | Silty gravels, gravel-sand-silt mixtures |
| | | | GC | Clayey gravels, gravel-sand-clay mixtures |
| | Sands | Clean sands (Little or no fines) | SW | Well-graded sands, gravelly sands, little or no fines |
| | | | SP | Poorly graded sands, gravelly sands, little or no fines |
| | | Sands with fines (Appreciable amount of fines) | SM | Silty Sands, sand-silt mixtures |
| | | | SC | Clayey sands, sand-clay mixtures |

| Major Divisions | | Group Symbols | Typical Names |
|---|---|---|---|
| Fine-grained soils | Silts and Clays (Liquid limit less than 50) | ML | Inorganic silts and very find sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity |
| | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| | | OL | Organic silts and organic silty clays of low plasticity |
| | Silts and Clays (Liquid limit greater than 50) | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts |
| | | CH | Inorganic clays of high plasticity, fat clays |
| | | OH | Organic clays of medium to high plasticity, organic silts |
| | Highly Organic Soils | Pt | Peat and other highly organic soils |

UNIFIED SOIL CLASSIFICATION SYSTEM

**CONSISTENCY CLASSIFICATION FOR SOILS**

According to the Standard Penetration Test

| Penetration Index (Blows / Ft) | Granular | Cohesive |
|---|---|---|
| 0-4 | Very loose | Very soft |
| 5-9 | Loose | Soft |
| 10-19 | Slightly compact | Stiff |
| 20-34 | Compact | Very stiff |
| 35-69 | Dense | Hard |
| >70 | Very dense | Very hard |

**LEGEND OF EARTH MATERIALS**

- GRAVEL
- SAND
- SILT
- CLAY
- SANDY CLAY or CLAYEY SAND
- SANDY SILT or SILTY SAND
- SILTY CLAY
- CLAYEY SILT
- PEAT and/or ORGANIC MATTER
- FILL MATERIAL
- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis unless gradations and Unified Soil Classification are shown.

Blows per foot may be converted to approximate $\phi$ angle or approximate cohesion (C)

## LEGEND OF BORING OPERATIONS

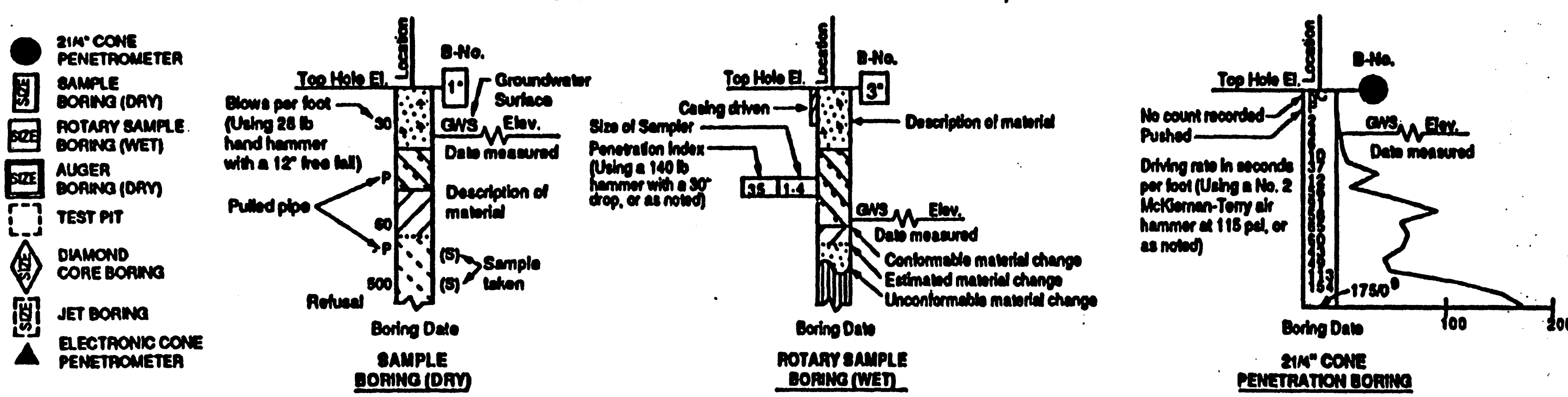
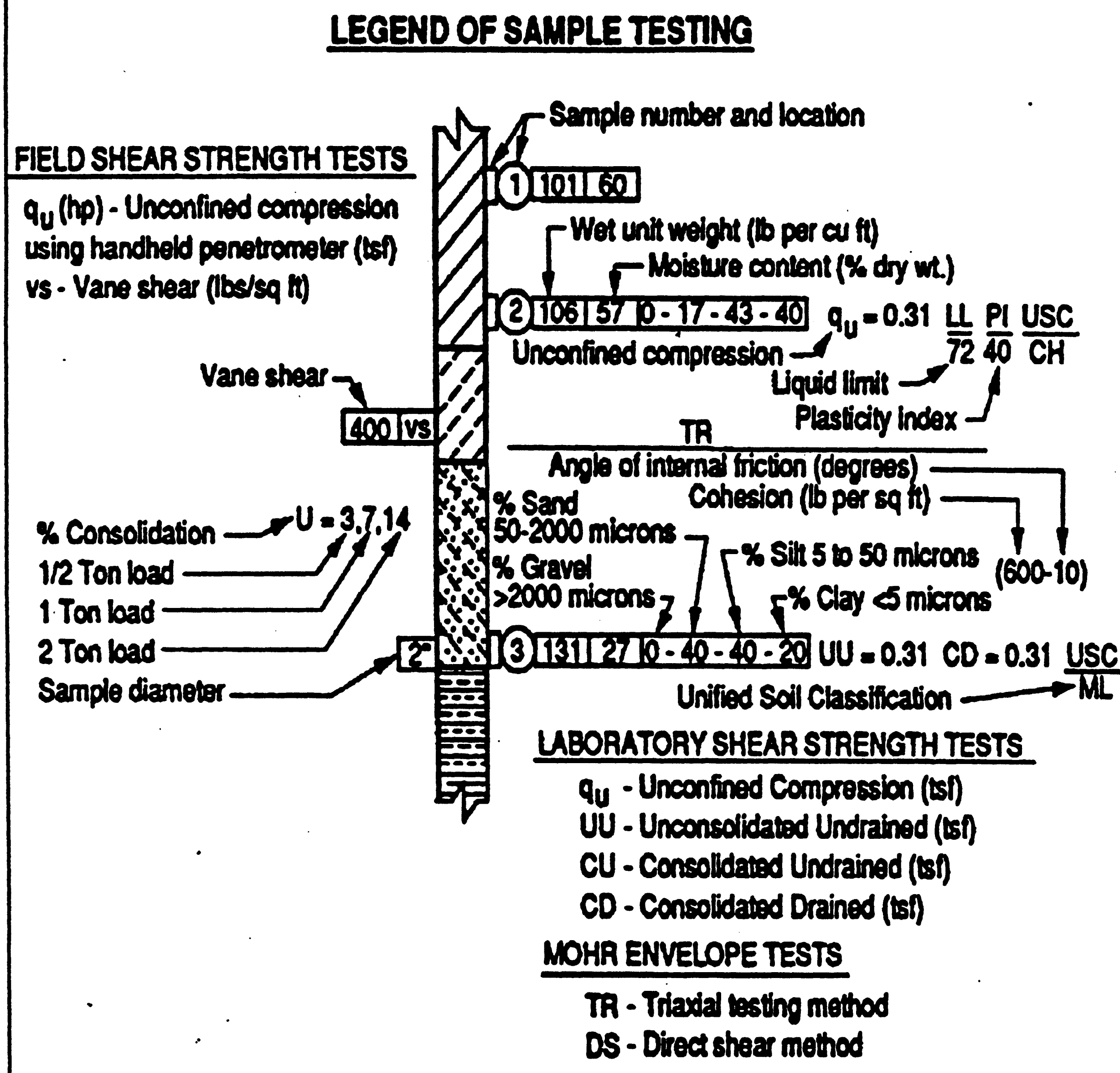



FIGURE 5




Example Boring Legend
As Used On State Plans

**FIGURE 6**

# UNIFIED SOIL CLASSIFICATION SYSTEM

| Inches | U.S.Standard Sieve No. | Particle Size |
|---|---|---|
| Over 8" | | BOULDER |
| 8" to 3" | | COBBLE |
| 3" to 3/4" | | GRAVEL Coarse |
| 3/4" to | 4 | GRAVEL Fine |
| | 4 TO 10 | SAND Coarse |
| | 10 TO 40 | SAND Medium |
| | 40 TO 200 | SAND Fine |
| | Under 200 | SILT or CLAY |

1 micron = 0.001 mm

**TABLE 10**

| USEFUL CONVERSIONS | |
|---|---|
| Pa = Pascal = $N/m^2$ | 1 $Ft^3$ = 0.028 $m^3$ |
| N= Newton | 1 $Ft^2$ = 0.093 $m^2$ |
| 1 ksf = 47.88 kPa = 47.88 $kN/m^2$ | 1 in = 0.025 m |
| 1 $Lb/Ft^2$ = 0.048 $kN/m^2$ = 47.9 $N/m^2$ | 1 $in^2$ = 645.16 $mm^2$ |
| 1 $Lb/in^2$ = 6.89 $kN/m^2$ | 1 kip = 4.448 kN |
| 1 $Ton/Ft^2$ = 2 ksf = 95.76 kPa | 1 kg = 9.807 N |

# SOIL CLASSIFICATION AND PROPERTIES

The Transportation Materials and Research Laboratory Engineering Geology Branch has prepared a summary of "simplified typical soil values" For average trench conditions the Engineer will find the data very useful to establish basic properties or evaluate data presented by the Contractor. The following table lists approximate values.

## SIMPLIFIED TYPICAL SOIL VALUES

| Soil Classification | $\phi$ Friction Angle of the Soil | Density or Consistency | $\gamma$ Soil Unit Weight (pcf) | $K_a$ | $K_w=K_a\gamma$ Equiv. Fluid Wt. (pcf) |
|---|---|---|---|---|---|
| Gravel, Gravel-Sand Mixture, Coarse sand | 41 | Dense | 130 | .21 | 27 |
| | 34 | Compact | 120 | .28 | 34 |
| | 29 | Loose | 90 | .35 | 32 |
| Medium Sand | 36 | Dense | 117 | .26 | 30 |
| | 31 | Compact | 110 | .32 | 35 |
| | 27 | Loose | 90 | .38 | 34 |
| Fine Sand | 31 | Dense | 117 | .32 | 37 |
| | 27 | Compact | 100 | .38 | 38 |
| | 25 | Loose | 85 | .41 | 34 |
| Fine Sand Silty, Sandy Silt | 29 | Dense | 117 | .35 | 41 |
| | 27 | Compact | 100 | .38 | 38 |
| | 25 | Loose | 85 | .41 | 34 |
| Silt | 27 | Dense | 120 | .38 | 45 |
| | 25 | Compact | 110 | .41 | 45 |
| | 23 | Loose | 85 | .44 | 37 |

TABLE 11

For active pressure conditions use a unit weight value of $\gamma$ = 115 psf minimum when insufficient soils data is known.

A rough correlation between the standard penetration index value (N) and the angle of internal friction ($\phi$) in a granular can be made as shown by TABLE 12. Also, the penetration index can be related to the cohesive value (C) in a cohesive soil as shown in TABLE 13. The standard penetration index is converted to $q_u$ (unconfinedcompressive strength) which in turn is equated to "C" by the formula, $C = q_u/2$.

Please note that these conversion tables are approximate. They can be used by characterizing the soil as being either predominately granular or cohesive. If possible, the conversion of the penetration index (N Value) should be checked by performing laboratory or in-site tests.

## GRANULAR SOILS

| COMPACTNESS | VERY LOOSE | | LOOSE | | MEDIUM | | DENSE | | VERY DENSE |
|---|---|---|---|---|---|---|---|---|---|
| Relative Density, $D_d$ | | 15% | | 35% | | 65% | | 85% | |
| Standard Penetration Resistance, N = Blows/ft* | | 4 | | 10 | | 30 | | 50 | |
| Angle of Internal Friction, $\phi$ | | 28 | | 30 | | 36 | | 41 | |
| Unit Weight (PCF) | | | | | | | | | |
| Moist | 100 | | 95-125 | | 110-130 | | 110-140 | | 130+ |
| Submerged | 60 | | 55-65 | | 60-70 | | 65-85 | | 75+ |

VERY LOOSE: A reinforcing rod can be pushed into soil several feet.
DENSE: Difficult to drive a 2x4 stake with a sledge hammer.

* N = Blows/Ft as measured by the standard penetration test (See Appendix B).

$$\text{Relative Density, } D_d = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

e = existing void ratio of mass being considered.
$e_{max}$ = void ratio of same mass in its loosest state.
$e_{min}$ = void ratio of same mass in its most compact state.

TABLE 12

# COHESIVE SOILS

| CONSISTENCY | VERY SOFT | | SOFT | | MEDIUM | | STIFF | | VERY STIFF | | HARD |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $q_u$ = unconfined comp. strength (PSF) | | 500 | | 1000 | | 2000 | | 4000 | | 8000 | |
| Standard Penetration Resistance, N = Blows/Ft * | | 2 | | 4 | | 8 | | 16 | | 32 | |
| Unit Weight (PCF) Saturated | | 100-120 | | | 110-130 | | | 120-140 | | | 130+ |

VERY SOFT: Exudes from between fingers when squeezed in hand.
SOFT: Molded by light finger pressure.
MEDIUM: Molded by strong finger pressure.
STIFF: Indent by thumb.
VERY STIFF: Indent by thumb nail.
HARD: Difficult to indent by thumb nail.

* N = Blows/Ft as measured by the standard penetration test (See Appendix B).

To be used only as a rough guide.

**TABLE 13**